Improving the seismic performance of existing reinforced concrete buildings using advanced rocking wall solutions

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ABSTRACT: Recent major earthquakes such as Northridge 1994 and Izmit Kocaeli 1999 highlighted the poor performance of existing buildings constructed prior to the early 1970's. Low lateral seismic design coefficients and the adopted "working stress design" methodology (essentially an elastic design) lacked any inelastic design considerations, thus leading to inadequate detailing. Insufficient development lengths, lapping within potential plastic hinge regions, lack, or total absence of joint transverse reinforcement, and the use of plain round reinforcement and hooked end anchorages were common throughout the structure. The behaviour is generally dominated by brittle local failure mechanisms (e.g. joint or element shear failures) as well as possible soft-storey mechanisms at a global level. Amongst several possible retrofit interventions, a typical solution is to provide the structure with additional structural walls i.e. external buttressing or column in-fills.

Extensive developments on precast, post-tensioned, dissipative systems have shown promise for the use of rocking wall systems to retrofit existing poorly detailed frame structures. In this contribution, the feasibility of such a retrofit intervention is investigated. A displacement-based retrofit procedure is developed and proposed, based on targeting pre-defined performance criteria, such as joint shear and/or column curvature deformation limits. A design example, using the proposed retrofit strategy on a prototype frame is presented. A brief overview on experimental work ongoing at the University of Canterbury investigating the dynamic response of advanced rocking walls for retrofit purposes will be provided.

1 INTRODUCTION

It has been well reported that the structural performance of pre 1970's buildings consists of brittle, premature local failure mechanisms leading to possible collapse of the lower or ground floors (Pampanin (2006)). Reasons for such a failure stem from low lateral design coefficients with an expectation that the building will remain elastic. The "elastic" analysis and design, or "working stress" concept, had no allowance for member ductility and hence the structure had insufficient detailing to properly accommodate the inelastic demand due to seismic loadings. Typically, the use of plain round reinforcing bars, lapping of longitudinal bars in potential plastic hinge regions, insufficient development lengths of reinforcing bars, and a lack or total absence of joint transverse reinforcement was common.

Following a brief overview into the performance of existing RC buildings and the advantage of using coupled walls for retrofit applications, the feasibility and efficiency of implementing a rocking/dissipating (PRESSS-technology) wall system is presented. A performance based (displacement-based) retrofit design methodology is developed and presented for poorly detailed (pre-1970's) frame buildings. An example of the design procedure is also given along with an update on the on-going shake-table testing at the University of Canterbury.

2 PERFORMANCE ASSESSMENT OF EXISTING BUILDINGS

Typical joint failure mechanisms for poorly detailed exterior beam-column joints without joint shear reinforcement are shown in

Figure 1 (Priestley (1997); Pampanin, et al. (2003)). Depending on the beam anchorage details different damage mechanisms can occur. When beam bars are bent into the joint (**Figure 1**a, **Figure 1**b) cracking will initiate from dilation of the un-reinforced joint core followed by opening of the hooked anchorages. A more brittle mechanism can be expected if the beam bars are bent away from the joint (**Figure 1**c) as the diagonal compression strut is not adequately "captured" at the anchorages' bent-end preventing the development of a "nodal" point for internal forces. Plain round bars with hooked end anchorages (**Figure 1**d) cause slipping of the reinforcement thought the joint resulting in stress concentrations at the anchorage ends leading to a wedge failure of the joint.

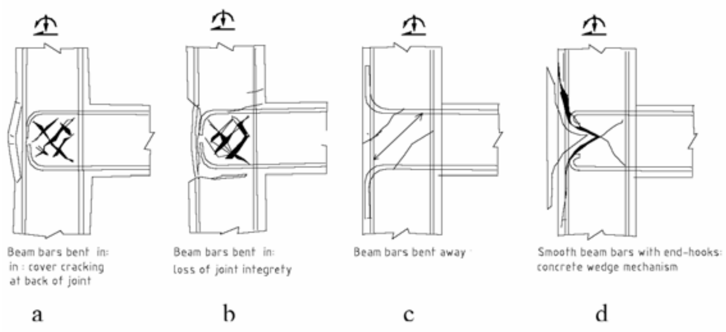


Figure 1: Typical anchorage details for beam-column joints Pampanin, et al. (2003)

Typical details within the pre-1970's building stock are shown in Figure 2a. Lapping within plastic hinge zones, insufficient development lengths, and the low level of transverse reinforcing content were common practice. The expected ductility capacity of these elements will be relatively low due to bar slip, poor confinement, buckling of longitudinal reinforcement and joint shear failure.

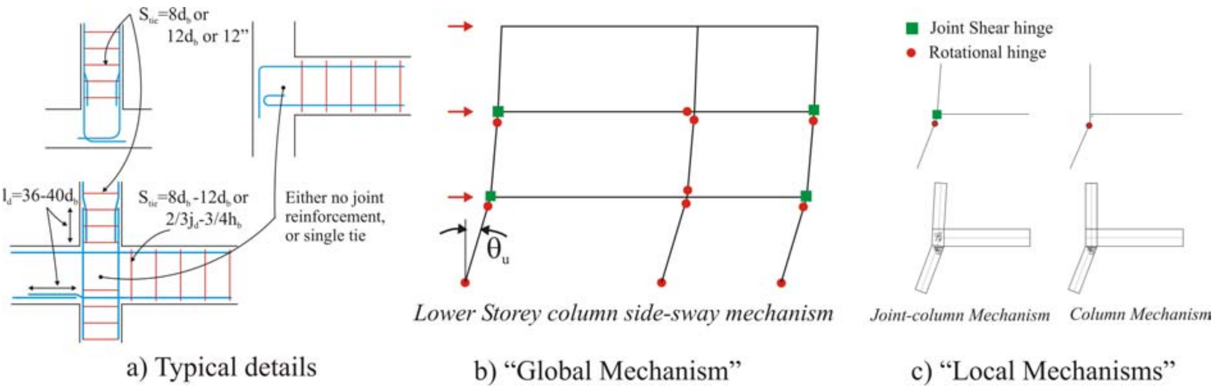


Figure 2: Failure mechanisms and typical NZ detailing

Figure 2b illustrates a soft-storey "global mechanism" for a 3-storey building. The independent, "local mechanisms" are also shown (Figure 2c). In defining the "local mechanism", a comparative strength assessment allows a hierarchy of strength and sequence of events to be evaluated within the beam-column joint subassembly. The capacity of each structural element (beam, column, and joint) is converted to a common-unit, such as an "equivalent column moment", allowing a direct strength comparison to be made within a moment-axial domain (M-N interaction diagram). Both flexure and shear failure modes can be considered as well as strength degradation effects.

2.1 Beam, column, and beam-column joint assessment

An extended literature on the assessment of existing reinforced concrete buildings is available and to some extent, regulated within code previsions e.g. fib (2003), FEMA:356 (2000), NZSEE (2006).

The performance of RC beams and columns are influenced by the type of reinforcing (plain round vs.

deformed), the presence of lap splices within the plastic hinge region, and the amount of transverse reinforcement for confinement, anti-buckling and shear resistance. Plain round bars result in slipping of the longitudinal reinforcement and poor energy dissipation, while lapping within plastic hinge regions will result in premature failure under repeated cyclic loading at low levels of ductility.

The strength and deformation capacity of beam-column joints are more delicate. Joint shear stress and deformation limits are typically provided in assessment guidelines FEMA:356 (2000). However, based on recent experimental investigations, principle tensile stresses have been suggested to be a better indication of damage. Furthermore, strength degradation curves (principal tensile stress vs. joint rotation) have been proposed in literature (Priestley (1997)). Depending on the structural detailing (bars bent into the joint, bars bent out of the joint, hooked anchorage etc), the typology (interior vs. exterior), and of the reinforcing type (plain round vs. deformed) the joint deformation limits can vary significantly (Pampanin, et al. (2003)).

3 FRAME-WALL COUPLED SYSTEMS FOR RETROFIT

Adding either internal or external wall systems is an attractive and common option to retrofit an existing poorly detailed RC frame structure. As a result of the coupling mechanism between the frame and wall, a reasonably stiff, strong system would result, reducing structural deformations (thus deformation related damage) to reasonable limits. The use of monolithic wall systems can however result in significant physical damage and residual deformations, especially when considering the effects of near field events i.e. large velocity pulses. Figure 3 indicates the qualitative performance of a retrofit intervention based on an advanced rocking/dissipating wall when compared to the use of a traditional monolithic wall retrofit. The re-centring capacity of the post-tensioned wall can be used to minimise residual deformations, where deformation of the wall is confined to a “controlled rocking” motion on the foundation, relieving the wall of any physical damage. In addition, more appropriate dissipation techniques can be applied to a rocking system where the advantage of both velocity proportional and displacement proportional devices for near and far field earthquake events respectively, can be utilised.

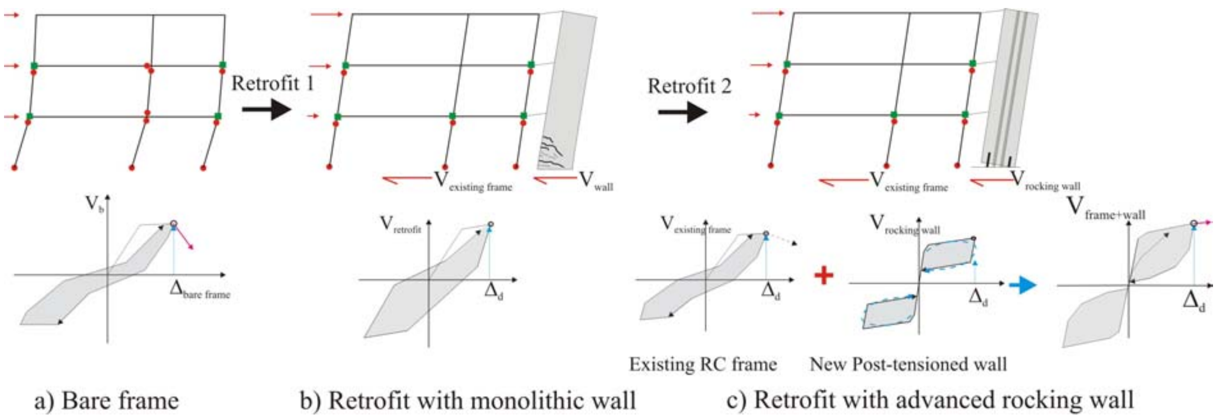


Figure 3: Comparison of retrofit intervention having either a monolithic wall or advanced rocking wall

A retrofit intervention based on the introduction of an additional wall system has the advantage of distributing the inelastic demand up the height of the structure, increasing the total strength, stiffness and energy dissipation of the existing bare frame.

The push-over capacity curve in Figure 4 summarises the key concepts for a retrofit intervention of the proposed prototype frame building. The first is that the deformation of the frame is reduced, limited to pre-determined deformation limits of critical elements within the frame. The second, the strength and deformation capacity of the wall alone are substantially improved by simply imposing the frame to deform linearly, redistributing the inelasticity (1); further improved by the additional over-turning capacity provided of the wall (2). Finally, additional energy dissipation (mild steel, friction, viscous, or post-tensioning alone) will provide a more damped structural response (3).

Special attention is required to a number of structural elements within dual systems due to a vertical displacement incompatibility between the frame and the wall (Paulay (1993)).

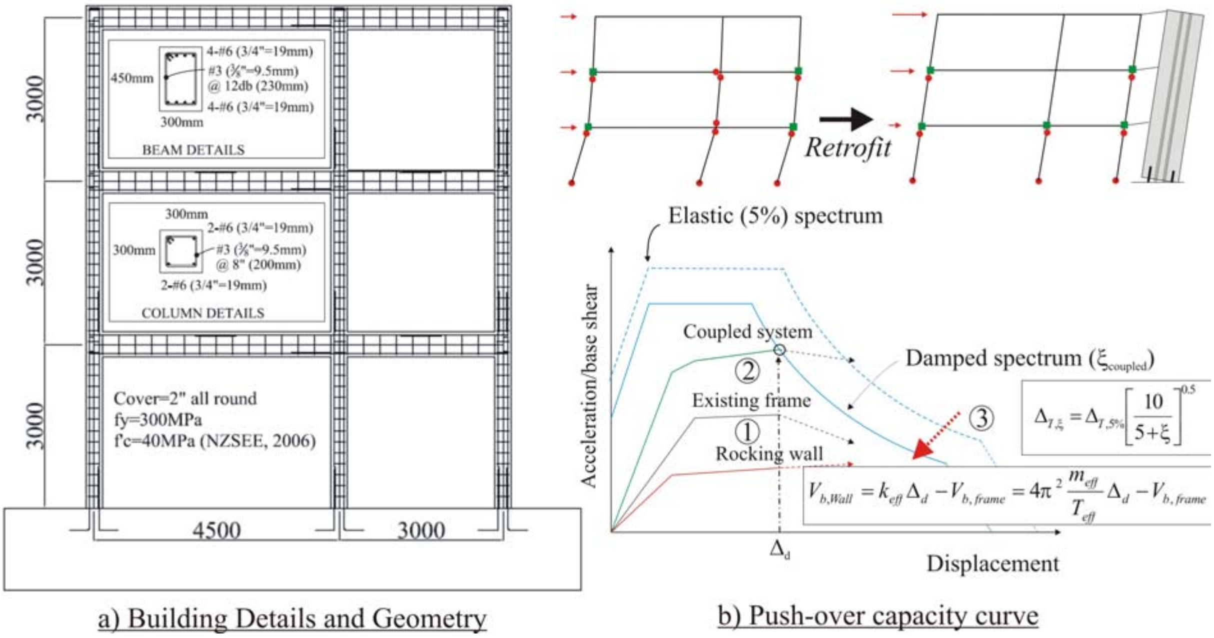


Figure 4: Prototype frame details

The vertical displacement arises from the shift in neutral axis position and the effect of having a "deep" member, resulting in lifting of the tension side of the wall and lowering of the compression side of the wall. This incompatibility can however be properly exploited to activate special yielding elements between two incompatible members (Kurama, et al. (2006)). Displacement-based design procedures have been recently developed for the design of new dual-system structures, including refined damping-ductility relationships (Sullivan, et al. (2006)).

4 ROCKING WALL SYSTEMS WITH ADVANCED DISSIPATION DEVICES

Jointed ductile rocking systems have been widely developed for either frame and wall systems (Priestley (2003), Rahman & Restrepo (2000), Kurama, et al. (2006)). A "controlled rocking" mechanism is developed at the critical interface (i.e. beam-column, pier-foundation interface, etc) and activates two types of reinforcement: prestressed tendons providing re-centring and non-prestressed mild steel dissipation (Figure 5). Rocking systems are characterised by reduced/negligible residual deformations, minimal physical damage (due to a single rotation located at the critical interface), whilst having similar maximum displacements when compared to their equivalently reinforced monolithic counterparts.

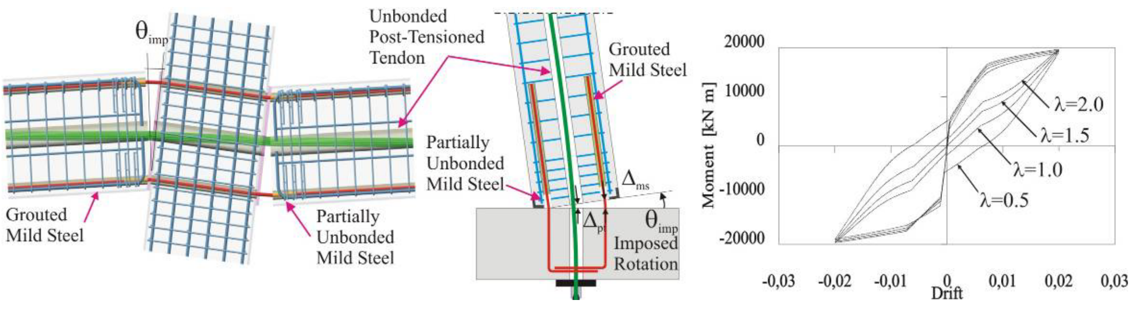


Figure 5: Rocking connection: Left, beam-column joint; Centre, pier/column-foundation connection; Right, cyclic behaviour as function of non-prestressed to prestressed moment ratio (λ).

The behaviour of a "hybrid system" (in terms of maximum and residual displacements) can be defined by a single "design parameter", lambda (λ) (fib (2004), NZS3101:2006 (2006)). Lambda is the moment ratio of the non-prestressed reinforcement (mild steel dissipation) to the prestressed reinforcement and/or axial load respectively, which defines both the energy dissipation and re-centring capacity of the connection/system (Figure 5).

Controlling structural deformations in existing or new buildings with supplementary dissipation has been extensively studied, including dampers ranging from metallic (elasto-plastic), viscous, visco-elastic, friction etc (FEMA:356 (2000), fib (2003)). More advanced materials include Shape Memory Alloys having "memory" characteristics suitable for use in seismic applications (Dolce, et al. (2000)).

5 PERFORMANCE BASED RETROFIT METHODOLOGY.

Performance-based design approaches are emerging as a more rational design and assessment procedure for the design of new structures and retrofit of existing buildings. Performance-based procedures generally relate performance to damage limit states considering allowable material strains and drift levels (including both maximum and residual). FEMA 356 provides a number of prescriptive performance criteria to be satisfied for the seismic rehabilitation of existing buildings. Indicative limit states, based on maximum and/or residual deformations, are suggested in order to satisfy various structural protection levels.

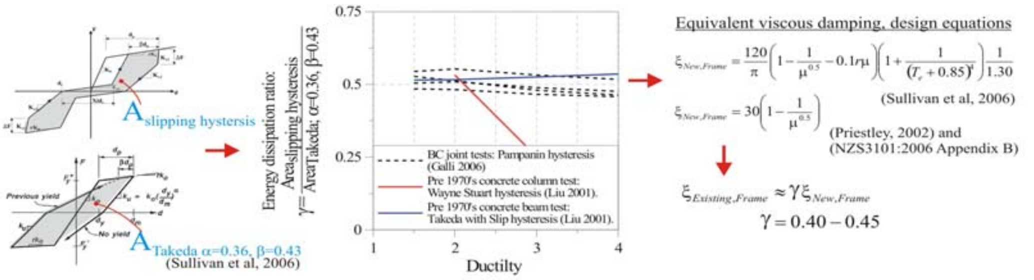
A displacement based design retrofit procedure is outlined as an extension of the Displacement Based Design Procedure (DDBD) proposed by Priestley (2002). Figure 7 summarises the basic steps within the procedure.

Step 1): A target displacement is defined based on allowable deformation limits of critical elements i.e. joint rotation or member curvatures.

Step 2): The retrofitted system, comprising of a coupled frame-wall (dual system) is converted to an equivalent single degree of freedom (SDOF) elastic system, with secant stiffness to the target displacement at the effective height Figure 7(a) and Figure 7(b).

Step 3): The damping of the coupled system (as built frame plus rocking wall) is evaluated based on a weighting in proportion to the base shear carried by each element Figure 7(c). Limited information is available on damping-ductility curves for poorly detailed buildings when joint damage mechanisms are activated. Equivalent viscous damping ratios in the order of 10% have been suggested by Priestley (1997). Damping-ductility relationships herein have been evaluated using stiffness degrading, pinched hysteresis rules based on previous analytical and experimental works (Galli; (2006), Liu; (2001), Chen; (2006), Sullivan, et al. (2006))

Figure 6 illustrates the ratio between the relative energy dissipation for each of the respective hysteresis rules i.e. slipping/pinching hysteresis vs. Takeda hysteresis. The effects of strength degradation have not been included. Based on this work, a percentage of damping assigned to the as-built frame during design appears to be justified by an amount 40-45% that of a well designed frame.



(a) Energy dissipation relationship for existing RC frame

(b) Modification factor for equivalent viscous damping equations

Figure 6: Modified damping relationships for existing buildings

On the other hand, evaluation of the equivalent viscous damping for the rocking wall can be calculated following the guidelines of Appendix B NZS3101 (2006). In principle, the damping is a function of the lambda (λ) ratio mentioned previously. In this case a reasonably high value of λ=1.4-2.0 should be used to guarantee full re-centring of the bare frame, while also counteracting strength degradation and

P-Delta effects within the frame.

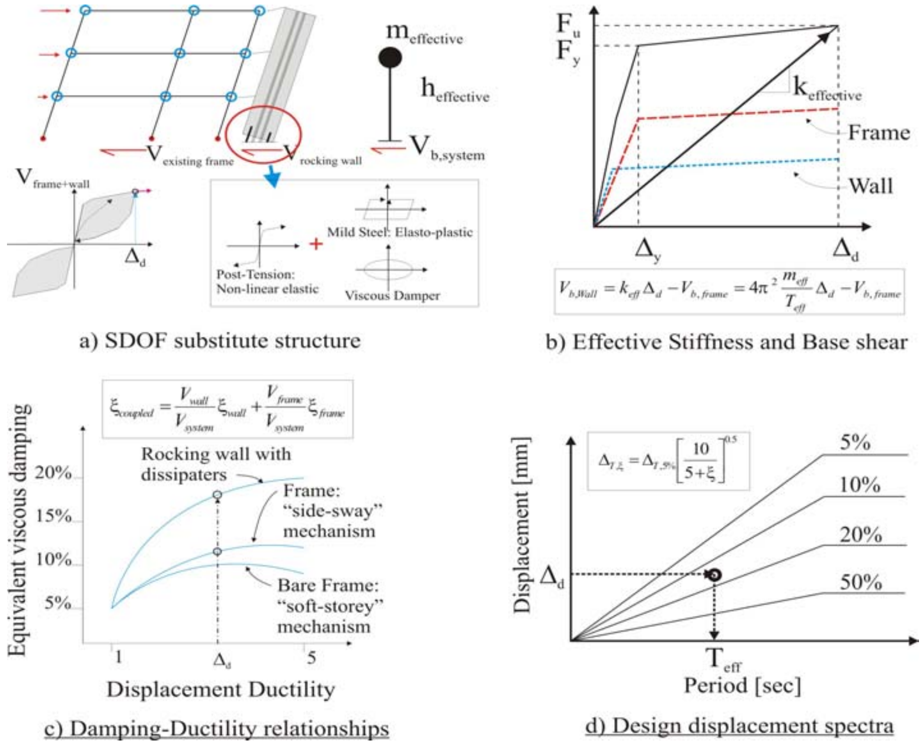


Figure 7: Displacement-based retrofit procedure

Step 4): The displacement response spectrum is used to deduce the effective secant period corresponding to the target displacement (Figure 7(d)). A reduction of the 5% elastic damped spectra is obtained by the η factor adopted in Eurocode 8 (EC8 (2003)).

Step 5): The total required base shear for the coupled system is calculated as $V_{b,total} = k_{eff} \cdot \Delta_d$. The base shear required of the hybrid wall for the retrofit intervention can thus be obtained as the difference between the total system base shear and base shear capacity of the bare frame.

6 NUMERICAL EXAMPLE: DDBD RETROFIT OF A PROTOTYPE BUILDING.

This section briefly outlines a numerical example based on a prototype building intended for testing at the University of Canterbury (Figure 4a). The three storey office building is located in Wellington (on shallow soil) where the retrofit intervention is to bring the capacity of the structure up to 100% of the current seismic loading requirements for new buildings.

6.1 Design procedure and results.

A "hierarchy of strength" assessment is performed on each joint within each beam-column joint at each storey level, allowing the overall base shear capacity of the bare frame to be determined (in this case, $V_{b,frame}$*=130kN (0.18g)*).

The deformation capacity of the frame is limited to the most critical elements within the as-built frame (Figure 7b). While the columns have only minimal transverse confinement, reducing their ductility capacity, the detailing of the joints would suggest to limit the allowable drift to 1.0% (limited ductility)

A yield drift of $\theta_y=0.80\%$ (corresponding to equivalent "joint yield", fib (2003)) results in a frame ductility demand of $\mu=1.3$. Equivalent viscous damping for the bare frame of $\zeta=6.3\%$ is based on a reduction of $\gamma=0.4$ to damping-ductility relationships presented Figure 6. The wall contributes $\zeta=11.9\%$, resulting in a total system damping of $\zeta=8.0\%$ (Figure 7c). The total system base shear is determined, defining the required wall capacity: $V_{b,wall}=58kN$, at a wall ductility of $\mu=5.0$ and moment ratio of $\lambda=1.4$. Table 1 summarises the numerical results of the DDBD retrofit example.

Table 1: Summary of displacement-based retrofit design example

Direct displacement-based design results			
θ_d	0.60%	$V_{b,frame}$	130 kN
Δ_d	68 mm	$\zeta_{e,frame}$	6.3%
h_e	6798 mm	$V_{b,Wall}$	58.0 kN
m_e	63016 kg	$\zeta_{e,Wall}$	11.9%
μ_{wall}	5.0	$V_{b,system}$	**188 kN**
μ_{frame}	1.3	ζ_e	**8.0%**
T_e	0.95 sec		
k_e	2762 kN/m		

Mechanism-Equivalent column moments	*Left*	*Center*	*Right*
Level 3: mechanism	Column	Column	Column
moment [kNm]	45	52	46
Level 2: mechanism	Joint	Joint	Joint
moment [kNm]	35	55	38
Level 1: mechanism	Joint	Joint	Joint
moment [kNm]	40	63	58
Ground: moment [kNm]	53	70	68

7 EXPERIMENTAL AND ANALYTICAL WORK ON ROCKING SYSTEMS.

An extensive experimental program at the University of Canterbury is currently investigating the seismic performance of jointed rocking systems with particular emphasis on bridge piers and wall structures. Quasi-static and pseudo-dynamic responses of 1/3-scale precast bridge piers with hybrid connections have been studied and modelled having either internal or external mild steel dissipation Marriott, et al. (2007). Dynamic shake-table testing of 1/3-scale wall specimens involving frequency constant, sinusoidal input motions are currently in progress. Both hysteretic (tension-compression yielding devices) and fluid-viscous (provided by FIP Industriale) are installed external to the wall section.

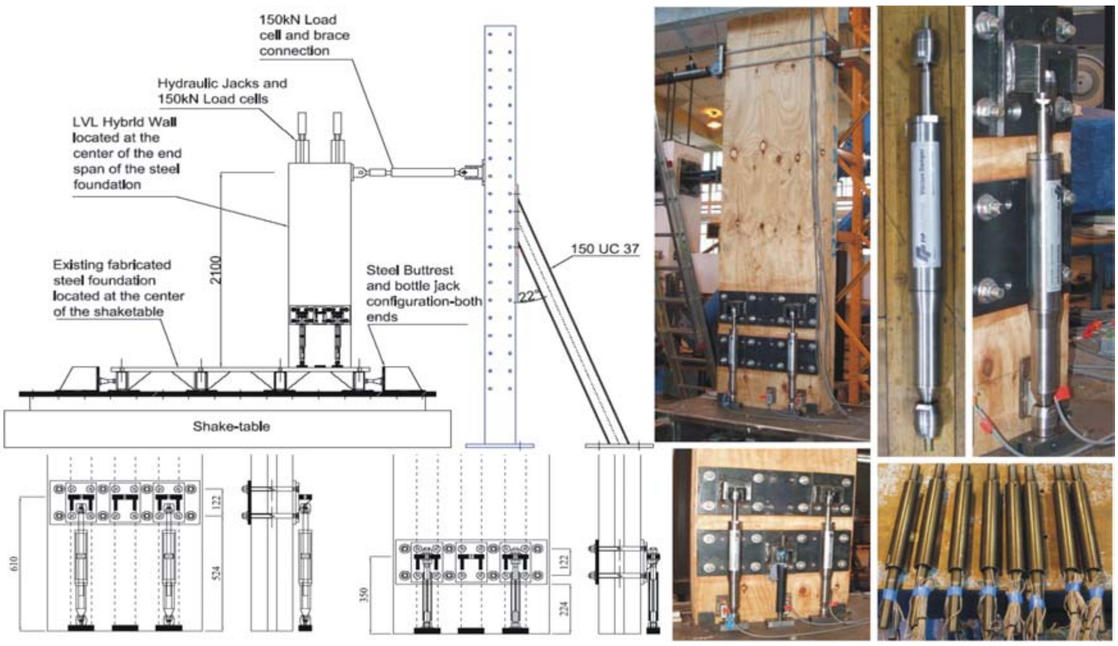


Figure 8: Dynamic wall testing images, Left: Experimental test set-up showing both viscous and mild steel damper arrangement; Right: Actual test rig and location of dissipation devices.

Figure 8 shows a number of images related to the experimental wall test. The studies will investigate the potential benefits of various types of dissipation (hysteretic, viscous, friction etc) used either independently or in combination with respect to either near field (velocity pulse or fling type events) or far field earthquake events. Recent numerical studies on Advanced Flag Shape Systems (Kam, et al. (2006)) have demonstrated that a combination of viscous (velocity proportional dissipation) and hysteretic (displacement proportional dissipation) devices would ensure a superior level of performance in either near field or far field earthquake events.

8 CONCLUSIONS

A conceptual performance-based retrofit design procedure for existing buildings based on the use of a rocking/dissipating (hybrid) wall system has been presented. The rocking wall can add lateral strength and damping to the structural system while controlling the damage in the as-built frame. Global inter-storey drifts for the frame can be limited, in addition to having the inelastic demand distributed up the height of the frame. The displacement-based retrofit procedure targets a pre-defined displacement, limiting deformations within critical structural elements such as joint rotations or member curvatures. Furthermore, residual deformations can also be minimised due to the re-centring capacity of the rocking wall.

Extensive experimental and analytical work is on-going at the University of Canterbury to further investigate the potential benefits of using precast post-tensioned rocking systems for either new design or retrofit solutions. Different dissipation characteristics (displacement or velocity proportional) are adopted and properly combined to improve the structural performance for either far field or near field earthquake events.

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